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OF THE
IOWA INSTITUTE
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HYDRAULIC RESEARCH
1939-1940

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Edited by
J. W. HOWE

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FOREWORD

Bulletin 19, *Two Decades of Hydraulics at the University of Iowa*, presented in abstract form an essentially complete record of hydraulic researches by staff members, students, and cooperative organizations associated with the Iowa Institute of Hydraulic Research from 1919, when the first unit of the hydraulic laboratory was constructed, through 1938. The present report reviews all graduate theses, publications of staff members, and work of associated agencies completed in the years 1939 and 1940. While much of the material is in abstract form, loan copies of all theses are available, and the master sheets from which the theses were reproduced are on file. Full reference is given to material which has been published by staff members during this period.

As a new feature of this series, four short papers by staff members are presented in full.

J. W. HOWE, Editor.

NEED FOR STANDARDIZATION OF TERMS USED IN STUDIES OF THE TRANSPORTATION AND DE- POSITION OF SEDIMENT

by

E. W. LANE

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As any part of the field of science develops, the terminology used in it evolves, becoming more extensive and more exact in its meaning. This is not an incidental consequence of the evolution, but a necessary and essential part of the process; and, within limits, the progress can be accelerated by assisting in the development of the terminology. The science of the transportation and deposition of sediment is over two centuries old, but most of the progress has been made in the last fifteen years. Along with this growth there has been an expansion of the terminology, but there is need of a still further extension of it and, particularly, of more precise definitions. No doubt it would be possible to carry efforts in this direction too far and thereby obstruct, rather than foster progress; but it is believed that the science has developed to a point where judicious efforts in this direction will bring forth very beneficial results.

One of the terms which needs to be standardized is the general term for the materials transported and deposited. In the engineering field this is usually concerned with mineral particles moved by water. In the geological field it includes also material moved by wind and glaciers. In the engineering field "silt" and "solids" have been used in this general sense. The term "silt" is objectionable, however, since in earth mechanics, agricultural soil studies, and in geology, the term is also used to designate a fine material composed of particles of a certain size range. The term "solids" is better, but sediment problems are concerned only with the transportation and deposition of the non-floating solids, and do not deal with floating ones such as ice and drift. It is believed that the solution of this matter is already on the way, in the broadening by the geologists of the term "sediment" to include not only material

deposited by water but also material being transported. The New Standard Dictionary (1937) gives the following definition of sediment: "Fragmental material transported by, suspended in, or deposited by water or air, or accumulated in beds by other natural agents; any detrital accumulation, such as loess." This definition has been adopted by some engineers and engineering organizations, but any acceleration of its adoption by other engineers would be a forward step.

An area in which there is great confusion is that of definitions of the various modes of transportation. It was early recognized that there are three main ways in which sediment is transported by a fluid. As early as 1848 these were described by Baumgarten.¹ They have also been pointed out by MacMath² and by Gilbert.³ According to these authorities, sediment is moved by: (1) rolling or sliding, (2) discontinuous hopping or bouncing and (3) suspension. The common term "suspension" so well fits the action of the third of these forms of movement that its use for this purpose has been universally adopted. For the hopping or bouncing stage, the term "saltation" was introduced by McGee⁴ and was adopted by Gilbert, to whom it is usually ascribed. This term also has been generally accepted. A definite name does not seem to be given to the rolling and sliding process. Gilbert did not directly give a name to the rolling and sliding portion of the load. Since some engineers believe that these three main forms of movement follow different laws, it is desirable to have a convenient, explicit name for this type of movement also. Even if further studies show that separate laws do not apply to the three types of motion, a definite term for this form of motion would still be desirable. For those cases where sliding is negligible the movement could be spoken of as "rolling," or, if it was desired to include both, "rolling-sliding" might be used. Another possibility is to use "in contact" or "contact" load, signifying that the motion is in nearly continuous contact with the bottom.

¹ Baumgarten, M., "Sur la portion de la Garonne qui s'étend en aval de l'embouchure du Lot dans le département de Lot-et-Garonne, et sur les travaux qui y ont été exécutés de 1836 à 1847." *Annales des Ponts et Chaussées*. p. 1-157, 1848-2.

² MacMath, R. E., "Silt Movement by the Mississippi — Its Volume, Cause and Conditions." *Journal of the Assoc. Eng. Soc.* Vol. 1, p. 268, 1881-1882.

³ Gilbert, G. K., "The Transportation of Debris by Running Water." *U. S. Geological Survey Prof. Paper 86*.

⁴ McGee, W. J., "Outlines of Hydrology." *Geol. Soc. of America, Bulletin* No. 19. p. 199, 1908.

Gilbert grouped all transportation into two forms: one, called "traction," included sliding, rolling, and saltation, and the other, "suspension." This is a useful combination and "traction" can well be adopted into the terminology.

Probably the least definite of the terms used in the field of sediment transportation and deposition is "bed load" as it is used with many different meanings by the various authorities. Kramer,⁵ in describing his experiment on the movement of material along the bed of a flume, used it as synonymous with the German word "Geschiebe" but did not explicitly define either. Straub⁶ used it extensively, interchangeably with "detritus," for the heavy material on or near the bed of streams, but also did not define it. O'Brien⁷ defined it as "material carried by a stream or channel in such a manner as to be caught in a trap of reasonable length and proper design." Einstein, Anderson, and Johnson⁸ suggested that bed load be defined as "that part of the total sediment load composed of all particles greater than a limiting size, whether moving on the bed or in suspension and including all bed material in movement." This limit is usually the lower size limit of that material for which the rate of transportation is a function of the stream discharge. It is also the lower limit of the sizes of material found in the bed in appreciable quantities. Recently, in a paper by Dr. Einstein,⁹ it was defined as follows: "Bed load is considered as the motion of bed particles in quick steps with comparatively long intermediate periods of rest. Thus bed load movement is a slow down-stream motion of a certain top layer of the bed." Certainly, with all these conflicting uses of the term "bed load" there is need of study of the possibilities of classification and standardization.

Some years ago the term "geschiebe" was introduced into American engineering practice from the German. This term means "that which is shoved." Its meaning is synonymous with "bed load." Since it failed to come into general use, although there are many

⁵ Kramer, Hans, "Sand Mixtures and Sand Movement in Fluvial Models." *Trans. A. S. C. E.* Vol. 100, p. 798, 1935.

⁶ "Missouri River." *House Document 238, 73rd Congress, 2nd Session*

⁷ O'Brien, M. P. and Rindlaub, B. D., "Transportation of Bed Load by Streams." *Trans. Amer. Geo. Union*, Vol. 15, Part II, p. 597, 1934.

⁸ Einstein, H. A.; Anderson, A. G.; and Johnson, J. W., "A Distinction between Bed-Load and Suspended Load in Natural Streams." *Transactions, A. G. U.* Part II, p. 632, 1940.

⁹ Einstein, H. A., "Formulas for the Transportation of Bed Load." *Proc. A. S. C. E.* Vol. 67, No. 3, p. 353, March, 1941.

points in its favor, it would probably be unprofitable to try to reintroduce it.

In order to open up a discussion which it is hoped will lead to a more explicit meaning of the term "bed load," the author would like to have consideration given to the following general definitions: Bed load is the coarse sediment which is being moved on or near the bed. Bed load would therefore include the sliding, rolling, and saltation load plus the coarser portions of the suspended load, if any is present. It is believed that this definition is in substantial agreement with the more extensive use of the term. Since this proposed definition of "bed load" includes in it part of the suspended load, the remainder of the load cannot be called "suspended load," and it is recommended that the term "non-bed load" be used.

There is also need of a name for the type of material included in the Einstein-Anderson-Johnson definition of bed load, since it is quite well established that there is a division of the load of many streams into two portions, one portion composed of sizes readily available in the bed and carried in quantities proportional to the discharge and the other portion composed of finer material, not readily available in the bed, the discharge of which is independent of the stream flow. The first class is what Einstein-Anderson-Johnson included in "bed load" and the latter they called "wash load." It is believed that it would be better to use the term "bed-material load" for the portion which they called "bed load" but to adopt their term "wash load" for the other portion. The use of the term "bed material load" gives an exact and self-explanatory term for this part of the load and eliminates the anomaly of calling a material "bed load," a considerable proportion of which may be carried in the upper half of the flow.

The use of the terms "bed-load sampler" and "suspended-load sampler" to designate the two types of sediment samplers is probably too well established to attempt to change, but it is apt to be somewhat in conflict with any definition of bed and suspended load which may be adopted, unless bed and suspended load are defined as material caught in bed- and suspended-load samplers, respectively. The material collected in a bed-load sampler is dependent on the type of sampler and the materials used in its construction and, therefore, with any other definition such an instrument will sample all the bed load only by coincidence. Since it probably will never be possible to separate particles moving in saltation from those in

suspension, a so-called suspended load sampler will sample both in the rare cases when it takes a sample in a zone where saltation load is moving. If these limitations of all forms of sampler are kept in mind, no great difficulty should arise in using the current terms for the classification of these sampling instruments.

Another need for definition or standardization is in the terms for expressing the amount (weight or volume) of material transported by a stream in a unit time. Gilbert used the term "load" in this sense. According to him one might say for example: "the load of the stream is ten pounds per second." Gilbert did not use the term "bed load" or "suspended load," but these are now so generally used that it is unlikely that they will be abandoned. The current use is to attach the word "load" to "bed," "suspended," "saltation," "traction," etc. to designate the material being transported in that specific manner. When it is necessary to designate the amount carried per unit time a word such as "discharge" would have to be added, as for example, "sediment discharge," "bed-load discharge." In view of the widespread use of the term "load" with other meanings, it is suggested that the word "discharge" be used when the amount of material transported per unit time is meant.

Summarizing, the author would recommend the term sediment for the general classification of non-floating material moved by a fluid and would recognize four types of movement of sediment: sliding, rolling, saltation, and suspension. The first two could be combined as "rolling-sliding" or "contact" and the first three as "traction." "Bed load" would be defined as coarse sediment in motion on or near the bed. "Bed-material load" would be defined as the sediment load composed of sizes readily available in the bed, and "wash load" as that composed of sizes not readily available in the bed. The total load could thus be divided into parts in four different ways as follows: (1) rolling-sliding or contact load, saltation load, and suspended load; (2) traction load and suspended load; (3) bed load and non-bed load; and (4) bed-material load and wash load. The word "load" should be used only in referring to a material in motion and the word "discharge" should be used in referring to the amount of material moved in terms of volume or weight per unit time.

SUSPENSION OF SEDIMENT IN UPWARD FLOW

by

HUNTER ROUSE

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Engineers have long been aware of the fact that the percolation of a fluid through a granular material will displace the material in the direction of flow if restraining forces are not sufficiently great. The quicksand phenomenon is a well known instance of such conditions as found in nature, while the method of washing rapid-sand filters by reversing the normal direction of flow represents an engineering application of the same principle; in either case the drag exerted by upward flow through an initially compact bed tends to exceed the immersed weight of the bed material, thereby producing an increase in bed porosity and a corresponding reduction in drag. Although in such examples of upward flow the material is rarely expanded to a point at which the grains are no longer in mutual contact, it is not unreasonable to presume that ever higher degrees of expansion would be obtained with increasingly higher rates of flow — the particles ultimately becoming so dispersed as to be in a state of complete suspension within the rising fluid. It is with the analysis of this phase of the phenomenon that the present paper is concerned.

Once bed material is displaced by such flow, it is apparent that the coefficient of permeability k in the Darcy relationship $Q = k A dh/dz$ will no longer equal that of the initially compact, uniformly mixed material, but will increase considerably as the interstices within the material grow in size. Moreover, as the expansion of the bed reduces the frictional restraint upon the individual particles, they will not only begin to go into suspension, but will at the same time assume positions commensurate with the resistance which they offer to the flow. In other words, finer or lighter particles will be carried higher than coarse or heavy ones, until a state of complete suspension displays almost perfect stratification of material according to size and density. If the upward flow is then abruptly

stopped, the material will settle to yield an even more perfectly stratified bed.

Since the Darcy coefficient k is at best an empirical factor, known values of which apply specifically to the case of pure percolation, a more rational approach to the case of complete suspension is to be preferred. Let it be assumed, therefore, that the immersed weight of the particles in suspension is exactly balanced by the drag exerted upon them by the upward flow. The external forces upon the elementary "free body" of the fluid-solid mixture shown in Fig. 1 must then also be in equilibrium, such forces involving the

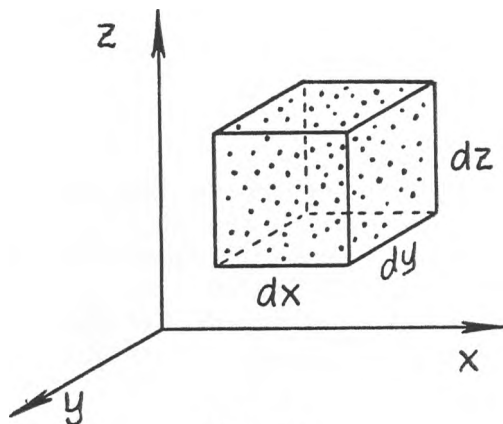


Figure 1.

weight of the mixture and the normal and tangential stresses upon the faces of the element. Designating by γ_s the specific weight of the solid matter, by γ the specific weight of the fluid itself, and by e the ratio of the volume of solids to volume of mixture, it follows that the weight of the element will have the magnitude $[e(\gamma_s - \gamma) + \gamma] dx dy dz$. If conditions are statistically similar at all points in any horizontal plane, the mean tangential stresses will be equal to zero over each of the faces. Normal stresses in the horizontal directions will then in themselves yield a state of equilibrium, while in the vertical direction the weight of the element will be balanced by the difference in pressure between the lower and upper faces. That is, expressing the change in pressure intensity with elevation as dp/dz , it will be seen that

$$- [e(\gamma_s - \gamma) + \gamma] dx dy dz - \frac{dp}{dz} dz (dx dy) = 0$$

whence

$$-\frac{dp}{dz} = \gamma + e(\gamma_s - \gamma)$$

Evidently, the rate of decrease of pressure intensity with elevation at any point is equal to the specific weight of the suspension at that point. Or, defining the concentration of the suspension as the difference between its specific gravity and that of the fluid alone (i.e., $c = e(\gamma_s/\gamma - 1)$),

$$-\frac{dp}{dz} = \gamma(1 + c)$$

In case the fluid is a liquid, the foregoing equation may be written in the more significant form

$$-\frac{dh}{dz} = c \quad (1)$$

in which the quantity h represents the manometric head $p/\gamma + z$ within the suspension—that is, the height to which the liquid would rise in an open manometer tube connected to a piezometer at the point in question. Eq. (1) therefore states that the rate of decrease of manometric head in the vertical direction is equal to the local concentration of the suspension. More practically expressed, the difference in manometric head between any two levels is directly proportional to the immersed weight of the material held in suspension between these two levels.

Although the foregoing relationship was obtained by assuming a state of static equilibrium to prevail between the immersed weight of the material and the drag exerted by the flow, this effectively presumes a kinematic balance between the settling velocity w of the material and the local velocity v of the upward flow. The settling velocity of the individual granules probably varies somewhat with the concentration of the suspension, but as a first approximation it may be considered proportional to the velocity of fall of a single representative particle in still liquid; thus, $w = K_1 v$. The local velocity of flow may likewise be expected to vary with concentration (or vice versa), but in a manner which is more readily evaluated. Representing by Q the rate of upward flow past a horizontal section of total area A , the local upward velocity should be directly proportional to the rate of flow and inversely proportional to the area of the reduced flow section. If the mean particle diameter is desig-

nated by d , and the mean spacing of the particles by L , the area of the total section may be expressed as $K L^2$ and that of the reduced section as $K (L^2 - K_2 d^2)$. Therefore, introducing the nominal velocity of flow $V = Q/A$ and combining the several proportionalities, it appears that

$$w = K_1 v = K_1 \frac{Q}{K(L^2 - K_2 d^2)} = K_1 \frac{KVL^2}{K(L^2 - K_2 d^2)} = \frac{K_1 V}{1 - K_2 \frac{d^2}{L^2}}$$

Similarly, the concentration c may be expressed in terms of the parameters d and L , in combination with the specific weights of fluid and solid, through the following proportionality:

$$c = e \left(\frac{\gamma_s}{\gamma} - 1 \right) = K_3 \frac{d^3}{L^3} \left(\frac{\gamma_s}{\gamma} - 1 \right)$$

Therefore,

$$\frac{d^2}{L^2} = \left(\frac{c}{K_3 \left(\frac{\gamma_s}{\gamma} - 1 \right)} \right)^{2/3}$$

Substitution of this expression in the foregoing relationship for w then yields the general equation

$$c = K_3 \left(\frac{\gamma_s}{\gamma} - 1 \right) \left(\frac{1 - K_1 V/w}{K_2} \right)^{3/2}$$

Finally, combining constants and introducing Eq. (1).

$$c = -\frac{dh}{dz} = A \left(1 - B \frac{V}{w} \right)^{3/2} \quad (2)$$

This equation is evidently in accord with the earlier statement that the finer material is carried toward the top of a zone of suspension, since c can decrease with elevation only if w also becomes smaller. Indeed, since the gradient dh/dz is a measure of c , one might now conclude that measurements of manometric head at various levels in such upward flow would provide a quantitative means of determining the fall-velocity characteristics of the suspended material. The import of such a conclusion cannot be over emphasized, for fall-velocity characteristics are comparable—and in some cases preferable—to size characteristics obtained by physical analysis. Certain features of the derivation, however, must not be overlooked. The coefficient A is governed by the shape and relative density of the material and perhaps even by viscous effects upon the pattern of flow around the individual particles; only if these factors are

essentially the same for all particle sizes may A be expected to retain a constant magnitude. The factor B , on the other hand, embodies the proportionality between the rate of settling and the local velocity of the upward flow; since turbulence can produce a state of suspension without any upward flow whatever, the magnitude of B must be expected to vary with the degree of turbulence present in the flow.

To what extent such influences will affect the practicability of this relationship can only be determined through experimental tests, a series of which were conducted at the University of Iowa in 1939-40 by Warren DeLapp and described in "Sediment Behavior in Upward Flow," a thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in the Department of Mechanics and Hydraulics. Duplicate studies were made in a glass cylinder 12 in. in diameter and 19 in. high and in a Lucite cylinder $1\frac{3}{4}$ in. in diameter and 54 in. high, each provided with means of measuring the manometric head over practically the entire height of cylinder. The granular material investigated consisted of quartz sand artificially graded to yield a size-frequency distribution closely following the normal error curve, 2% being finer than 0.07 mm. and 2% coarser than 0.8 mm.

A typical series of measurements made in the larger cylinder is shown in Fig. 2, the ordinate scale representing elevation above the fine-mesh screen which served to support the material at zero discharge, and the abscissa scale the difference between the manometric head within the water-sediment mixture and the head within the clear water above. Prior to the beginning of manometer readings, the entire bed was brought into suspension, and thereafter allowed to settle in stratified form. Measurements at a very low rate of flow then yielded the bottommost curve, the form of which is typical of pure percolation through a bed of decreasing grain size — that is, the Darcy permeability k grows smaller as the interstices are reduced in size, the slope — dh/dz attaining its maximum value at the bed surface. At the next higher rate of flow, however, a reversal in curvature is noted in this region, indicating that the topmost material has been carried into suspension. Continued increase in the discharge brings more and more material into suspension, at the same time increasing the drop in head through that portion of the bed which has not yet begun to expand. Finally, however, the form of the curve indicates that the entire bed has been carried into sus-

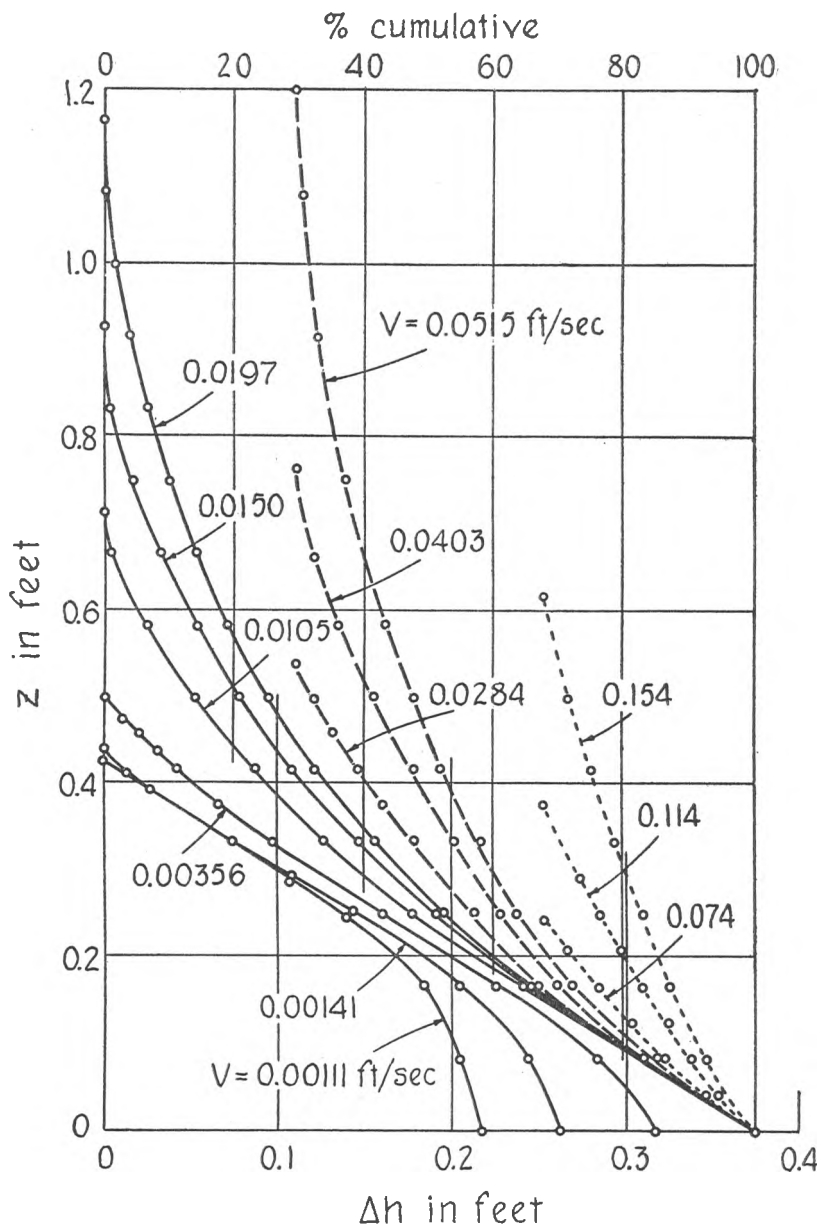


Figure 2.

pension, further increase in the rate of flow producing no further change in the head at the base of the column. This is in complete accord with Eq. (1), for the entire weight of the material is now supported by the upward flow. Indeed, supplementary tests proved that the addition or removal of known amounts of material invariably changed the manometer reading by the amount computed on the basis of Eq. (1). For this reason the broken curves in Fig. 2, obtained at such high rates of flow that the finer material had to be removed to avoid being carried over the top of the container, have arbitrarily been displaced to make the readings at zero elevation coincide.

From samples withdrawn at various elevations and at various rates of flow, it was possible to determine, by timing the fall in still water of a hundred or more representative particles of each sample, characteristic values of the quantity w appearing in Eq. (2). Measurement of the slope of $h:z$ curves at points represented by the

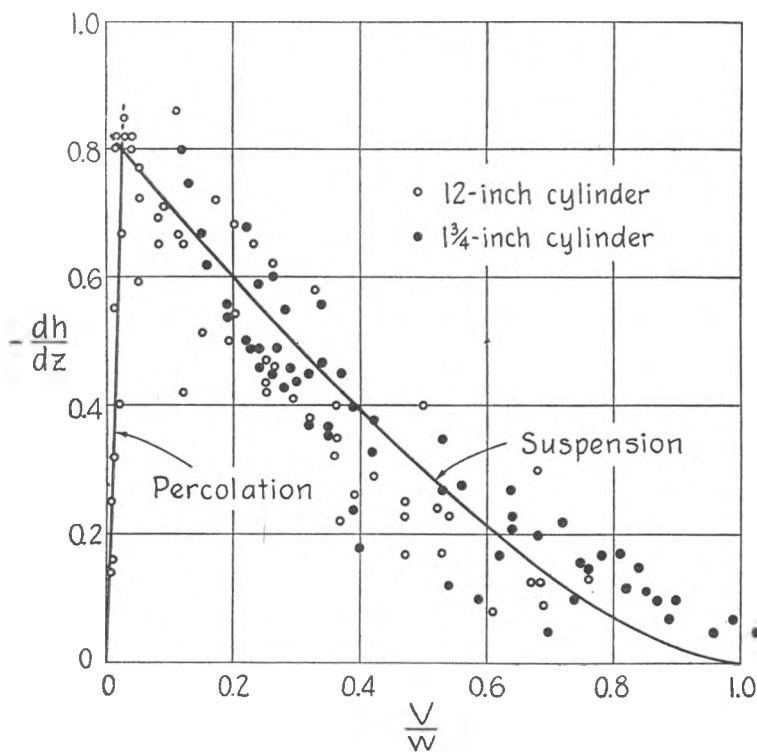


Figure 3.

samples then yielded the corresponding gradient $-dh/dz$, a plot of which against the ratio V/w is shown in Fig. 3. The straight line passing through the origin averages points obtained under conditions of pure percolation, its upper limit indicating that expansion of the material begins at a value of $-dh/dz$ of about 0.8, regardless of grain size. Once the material is in suspension, however, this gradient rapidly decreases in magnitude, finally approaching the limit zero as the nominal velocity of flow approaches the normal settling velocity of each different size of grain. Although there is appreciable scatter of points, they follow the general trend of the plotted curve (refer to Eq. (2)), for which $A = 0.83$ and $B = 1.0$. Nevertheless, it was found that the points lying above this curve invariably corresponded to the coarser material, and points below to the finer; in other words, the coefficient B , arbitrarily chosen as unity, actually varies with the fall velocity of the various grain sizes. Reference to the derivation of Eq. (2) will show that this indicates incomplete proportionality between w and v , a variation attributed to the effects of turbulence upon the suspension. Indeed, a definite pattern of eddies was noticeable through the transparent walls of the cylinders, apparently due neither to imperfect stilling of the approaching flow nor to the wake behind each individual

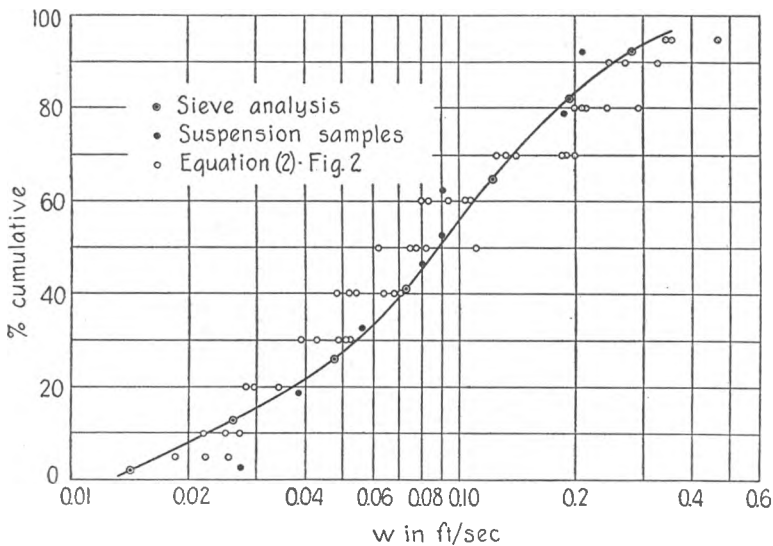


Figure 4.

grain, but rather to characteristic momentary fluctuations in the density of the water-sediment mixture.

Despite the probable lack of precision which such tendencies would produce if the method were used for sediment analysis, an effort was made to compare the results so obtained with the known characteristics of the original material. Using the size of sieve openings as a measure of the grain diameter and assuming the grains to be spherical in shape, points on the cumulative fall-velocity curve shown in Fig. 4 were computed. A second series of values was obtained by correlating the measured fall velocities of the several suspension samples with the corresponding points on the cumulative scale of Fig. 2. The third series resulted from measuring the slope $-dh/dz = c$ of the various curves of Fig. 2 for successive cumulative values, the corresponding magnitudes of w being obtained from the following form of Eq. (2).

$$w = \frac{V}{1 - 1.13 \left(-\frac{dh}{dz} \right)^{2/3}}$$

Although considerable deviation of the points from one another is again evident, the consistent trend of the measurements as a whole indicates that Eq. (2) embodies the basic principles of sediment suspension in upward flow.

Whether or not further investigation of the method will yield means of predicting quantitatively the effect of turbulence upon the sediment distribution, the use of upward flow as a means of segregating the various sediment grades is of immediate value. Indeed, the method has already been found effective in the preparation of highly sorted separates for use in other phases of sediment research, the desired grade being removed by siphon during settlement after the flow has been stopped. A somewhat similar procedure suggests itself as a possible improvement upon such methods of physical analysis as those of Wiegner and Crowther, based upon the theory of Odén, in which the sediment is first brought into a uniform state of suspension by thorough mixing of the entire vertical column of fluid. In other words, segregation by upward flow would replace the initial stirring process, and the w -frequency distribution would then be obtained from the temporal variation in manometric head near the base of the column during settlement of the material as a series of layers graded from the outset according to velocity of fall.

ADDITIONAL REMARKS ON FUNCTIONAL DESIGN OF FLOOD CONTROL RESERVOIRS

By C. J. POSEY AND FU-TE I

Respectively Associate Professor of Hydraulics and Structural Engineering and Associate Engineer, Iowa Institute of Hydraulic Research; and Hydraulic Engineer for Chinese National Bank.

Certain relationships pointed out by M. Kindinger, one of the discussors of a recent paper by the writers,¹ make it possible to apply the authors' method to problems such as the filling and emptying of tanks, the design of highway culverts with pondage, and other cases where the discharge varies according to some power of the head other than one-half or three-halves. The method allows a direct determination of the volume of storage necessary to hold the maximum outflow rate within a given value. It is necessary that the following conditions obtain.

1. The discharge through the outlet at the time filling is started must be small, say less than one or two per cent of the maximum outflow.
2. The discharge through the outlet must be susceptible of accurate approximation by an equation of the form

$$o = C_o h^n$$

where o is the outlet discharge rate, h the head on the outlet measured to the water surface in the tank or reservoir, and C_o and n are empirical constants.

3. The volume stored in the tank or reservoir above the zero level for h must be susceptible of accurate approximation by the equation

$$S = C_s h^m$$

in which S represents the storage and C_s and m are empirical constants.

¹ Posey, C. J. and I, Fu-Te, "Functional Design of Flood Control Reservoirs." *Transactions, A. S. C. E.*, vol. 105, p. 1638, 1940.

4. The variation of the inflow into the tank, with respect to time, is such that it can be approximated by a constant inflow extending over an interval of time which is small compared with that which would be necessary for the outflow to reach equilibrium with the inflow.

Whether or not a given problem conforms to conditions 1, 2, and 3 is easily ascertained. Conformity with condition 4 is more difficult to determine, for a comparison involves complete solution of the problem by some other method. Much more leeway is permissible in condition 4, however, than in conditions 1, 2, and 3. In doubtful cases a check solution should be made by some method in which the actual pattern of the inflow is taken into account.

The first step in the solution is to determine the exponents n and m by plotting the data logarithmically. The exponents are the slopes of the straight lines approximating the plotted points, and the deviation of the line from the points measures the error of the approximations, revealing whether conditions 2 and 3 are satisfactorily met. The percentage errors over 80 or 90 per cent of the range should not exceed 5 per cent. The accuracy should be best at the higher values of h .

If the inflow is constant for a given length of time and then ceases, condition 4 is fully met. If the inflow varies, an equivalent uniform inflow must be found. This is most easily accomplished by plotting a mass curve of inflow. On this diagram the uniform inflow becomes a straight line which should be made to fit the curve as well as possible, consistent with representing the same value for the total volume of inflow.

The final steps in the solution are as follows: Determine the outflow ratio, x , which is the ratio of the maximum outflow rate to the uniform inflow rate.

Then

$$S = d \cdot IT$$

where S is the volume of storage required, IT is the total volume of the equivalent uniform inflow, and d , the detention ratio, is a function of x and $\frac{m}{n}$, as given in the following table. Table 2 of the

original paper is more convenient if $n = 0.5$ or 1.5 . The functions used in evaluating d are given in the appendix of the original paper.

VALUES OF THE DETENTION RATIO FOR VARIOUS VALUES OF

 x AND $\frac{m}{n}$

Values of $\frac{m}{n}$	$x=0.1$	$x=0.2$	$x=0.3$	$x=0.4$	$x=0.5$	$x=0.6$	$x=0.7$	$x=0.8$
.6	0.961	0.921	0.877	0.831	0.780	0.723	0.661	0.584
.8	0.955	0.907	0.858	0.805	0.748	0.686	0.618	0.536
1.0	0.949	0.896	0.841	0.783	0.722	0.656	0.583	0.498
1.2	0.945	0.887	0.828	0.766	0.700	0.632	0.555	0.467
1.4	0.941	0.880	0.817	0.752	0.684	0.611	0.532	0.444
1.6	0.938	0.874	0.808	0.740	0.669	0.594	0.514	0.425
1.8	0.935	0.869	0.800	0.731	0.657	0.581	0.499	0.409
2.0	0.933	0.864	0.794	0.722	0.647	0.569	0.486	0.395
2.3	0.930	0.856	0.786	0.711	0.634	0.554	0.470	0.379
2.6	0.928	0.853	0.779	0.702	0.623	0.541	0.456	0.365
3.0	0.925	0.848	0.771	0.692	0.611	0.528	0.441	0.349
3.5	0.922	0.842	0.763	0.682	0.600	0.515	0.426	0.334
4.0	0.920	0.839	0.757	0.674	0.590	0.504	0.415	0.322
5.0	0.916	0.832	0.748	0.662	0.575	0.488	0.397	0.303
6.0	0.914	0.828	0.741	0.654	0.565	0.476	0.384	0.290
8.0	0.911	0.822	0.732	0.642	0.551	0.460	0.366	0.271
10.0	0.909	0.818	0.726	0.635	0.542	0.450	0.355	0.258
12.0	0.908	0.815	0.722	0.630	0.536	0.442	0.347	0.250

HYDRAULICS OF VERTICAL DRAIN AND OVERFLOW PIPES

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This report covers a series of extensive experiments on the flow of water down vertical pipes which act either as drains or overflows. Keeping in mind practical applications, a drain pipe is one which is connected flush with a horizontal surface such as a roof or the bottom of a tank, and an overflow pipe is one which extends an appreciable distance into a tank or vat. The experimental work was concerned only with the hydraulic and pneumatic conditions when water entered the top of the vertical pipes under such low heads that air was drawn down into the pipes and the pipes did not flow full. The experimental work was conducted on smooth pipes having internal diameters of 0.485, 0.309, and 0.144 ft. Studies were made on 3 or 4 different lengths of each pipe size. An empirical head-discharge relationship was developed which correlated the data reasonably well. The chief effects of varying the length of the pipes were on the value of the critical head which caused the pipe to flow full, and on the amount of air sucked down with the water.

A. DISCUSSION OF GENERAL PROBLEM

The flow of water down vertical pipes with various types of inlets has been reported in the literature at various times. So-called shaft spillways for dams have been used; a detailed discussion of the design of such a spillway with a widely flared entrance was presented by Kurtz.¹ The so-called "drop-inlet" as applied to soil conservation structures was experimentally studied and reported on by Kessler.² The hydraulic and pneumatic phenomena he observed

¹ Kurtz, Ford, "Hydraulic Design of the Shaft Spillway for the Davis Bridge Dam, and Hydraulic Tests of Working Models." *Trans. Amer. Soc. of Civil Engineers*, Vol. 88, p. 1, 1925.

² Kessler, L. H., "Experimental Investigation of the Hydraulics of Drop Inlets and Spillways for Erosion Control Structures." *Bulletin University of Wisconsin Engineering Experiment Station* No. 80, 1934.

were generally similar to those observed in the investigations reported herein.

A very careful study of the hydraulics and pneumatics of a small (1-in.) vertical overflow pipe was made by Binnie.³ Some of his data will be correlated with those obtained in this study on larger sizes of pipes. His description of the manner in which the flow occurs and the accompanying noises is very apt.

In general the flow into and down both overflow and drain pipes, as previously defined, is quite similar. At low heads the flow approaches simple flow over a weir, circular in plan. Of course, the nappe is not aerated which causes a greatly increased flow as compared to simple sharp-crest weir flow. As the water flows down the vertical pipe, which in our investigations was open to the atmosphere at its lower end, large quantities of air are sucked down. The water tends to cling to the sides of the vertical pipe with the air in the center. The water is naturally filled with small bubbles of air but it does not break up into a spray. The maximum velocity of the falling water is reached within a few feet of the entrance; this was discussed in detail in a previous publication on this general problem by Dawson and Kalinske.⁴

The total rate of air flow down the pipe depends on the pipe size, water discharge, and length of pipe. The maximum rate of air flow is obtained at a relatively low water discharge, and this air flow in volume units may be in excess of the water flow. The ability of water flowing down a partly full vertical pipe to suck down a large quantity of air has been put to practical use. A detailed discussion of this is given by Peele⁵ in his book on the design of compressed air plants.

The studies described herein were concerned with vertical pipes having square cut edges at the entrance. Such installations of drain and overflow pipes are extremely common for roof rain leaders and tank drains and overflows of various types. The main purpose of the experiments was the developing of a head-discharge relationship and the determination of the effect of pipe length on such a relation-

³ Binnie, A. M., "The Use of a Vertical Pipe as an Overflow for a Large Tank," *Proc. Royal Society of London*, Vol. 168-A, p. 219, 1938.

⁴ Dawson, F. M. and Kalinske, A. A., "Report on Hydraulics and Pneumatics of Plumbing Drainage Systems," *University of Iowa Studies in Engineering*, Bull. No. 10, 1937.

⁵ Peele, R., "Compressed Air Plants," Wiley, p. 248, 1930.

ship. The amount of air drawn down by the flowing water was determined principally as a matter of general interest.

B. EXPERIMENTAL APPARATUS AND PROCEDURE

The entire experimental apparatus used in these studies is shown in Fig. 1. The first item of importance to note is the water-supply line and the design of the supply tank to insure radial flow into the pipe entrance. The rate of water flow was measured by either a 6" x 3" or 3" x 1½" venturi meter, the size used depending upon the rate of flow. Both meters were calibrated by means of large weighing tanks.

The height of the water in the tank was measured to the nearest thousandth of a foot on a glass manometer which was connected to the false bottom of the tank at several places about 18 in. from the

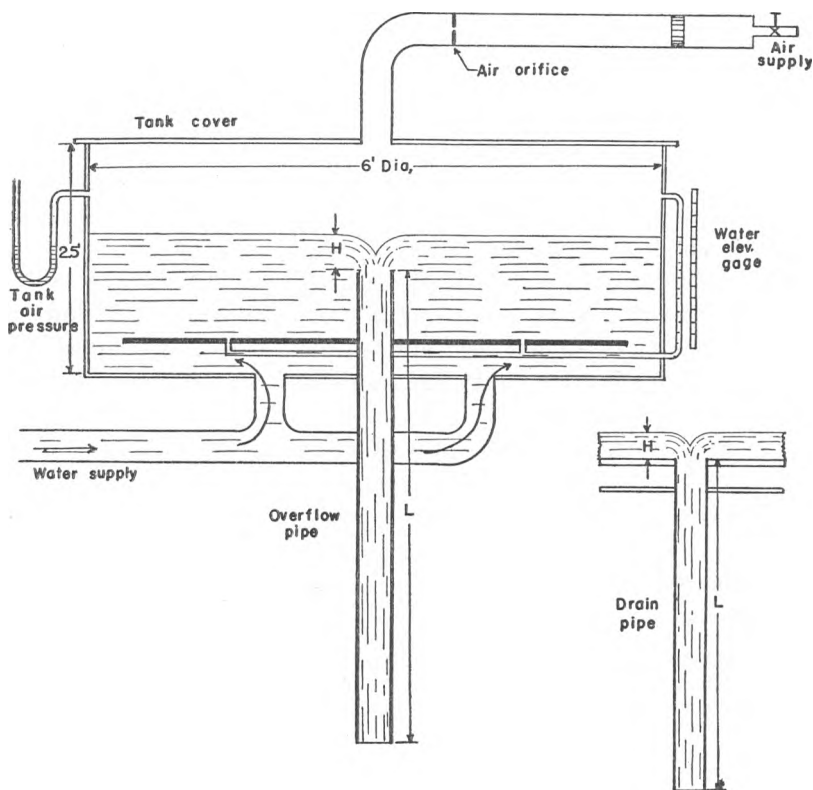


Fig. 1. The experimental apparatus.

center of the pipe entrance. The head H above the pipe entrance was determined from this reading.

The runs for obtaining data on the head-discharge relationship were taken with the top of the tank completely open to the atmosphere. The experiments were started with the smallest head it was convenient to use and the head was gradually increased until the pipe started to flow full.

The rate of air flow was determined in the following manner: The entire tank was tightly covered and the tank was connected to a separate air-supply line as shown in Fig. 1. The height of water in the tank was read by the same glass manometer as used previously except that the top of the gage was connected to the tank above the water line. A glass U-tube was used to determine the air pressure above the water inside of the tank. After setting a certain rate of water flow, air was supplied to the tank at such a rate as to maintain atmospheric pressure inside of the tank. The rate of air flow was measured by means of a standard orifice located in the air-supply line. The head difference across the orifice was kept below 4 in. of water in order that the thermodynamic corrections would not be necessary in calculating the air flow. This necessitated having a wide range of orifice sizes. This method of determining the air flow was found to be very satisfactory; care, however, had to be taken to maintain the air pressure inside of the tank as close to atmospheric pressure as possible since it was found that the rate of air flow was considerably affected by small pressure differences.

The vertical pipes used were made of transparent "lucite" and had a wall thickness of $\frac{1}{8}$ in. The transparent pipes were used to permit both visual observation of the flow and the recording on motion picture film of the various interesting phenomena.

C. RESULTS OBTAINED

Experiments were made on the 0.485 ft., the 0.309 ft., and the 0.144 ft. pipes in that order. Each pipe was first installed as a drain pipe, its top being flush with the tank bottom; then each pipe was installed so that it projected about 10 in. up into the tank, thus acting as an overflow pipe. A few runs were made for each size with the pipe projecting 5 in.; the results were similar to those for the 10 in. overflow. For each size of pipe and for each condition of entrance four different lengths were used, thus making a total of

8 separate test installations for each pipe size, or 24 in all. Sufficient water was not available to obtain heads high enough for the 0.485 ft. pipe to flow full. Therefore, the critical head was not determined for this pipe size.

The head-discharge data obtained are plotted in Figs. 2, 3, 4, 5, 6, and 7. Figs. 2, 4, and 6 show the data when the three pipes were used as overflow pipes, and Figs. 3, 5, and 7 when they were used as drain pipes. The most significant item to note is that all the data for any pipe size and entrance condition tend to fall on one smooth curve for all the pipe lengths as long as the pipe does not flow full. The head at which the pipes begin to flow full was found to depend on their length, the longer the pipe the greater being the head that is necessary above the pipe entrance to cause it to flow full.

The transition from partly full to completely full flow in the pipes is not a very definite phenomenon. After full flow is attained,

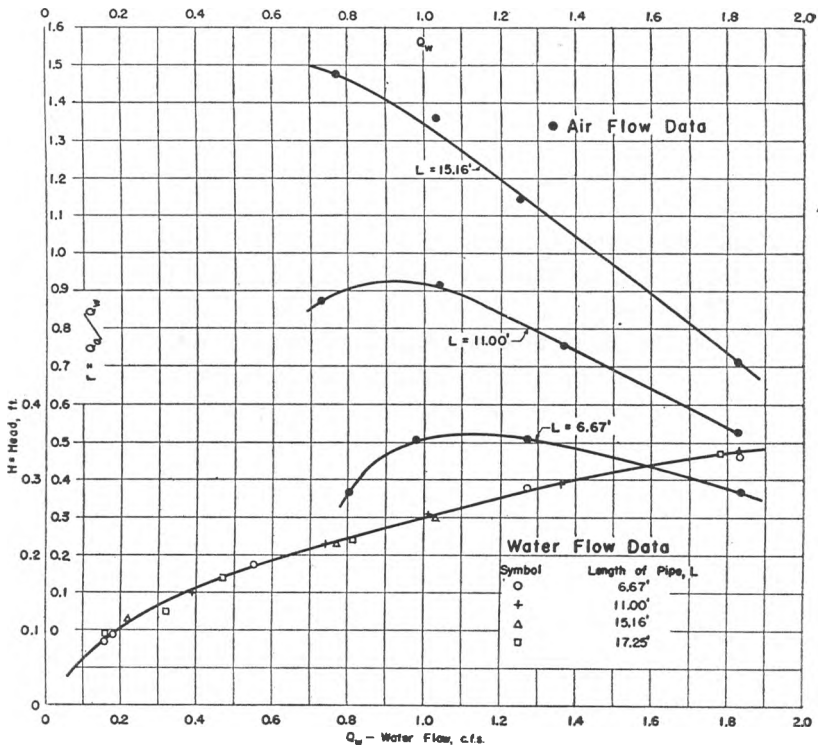


Fig. 2. Water and air flow data for 0.485 ft. overflow pipe.

at the lower heads there is occasional "gulping" of air. As soon as the pipe begins to flow full, any further increase in head above the pipe entrance does not appreciably increase the rate of discharge. In this region the head-discharge relationship is, of course, that for ordinary pipe flow with a sharp-edged or "reentrant" entrance condition.

Another interesting item to note is that for any given head in the region of partly full flow the overflow pipe discharges more water than the drain pipe. The head at which full flow starts for any given length of pipe appears to be slightly larger for the drain pipe — that is, with the end of the pipe even with the bottom of the tank.

The present experimental set-up did not permit longer pipes than those used; however, it would be of interest to obtain further data

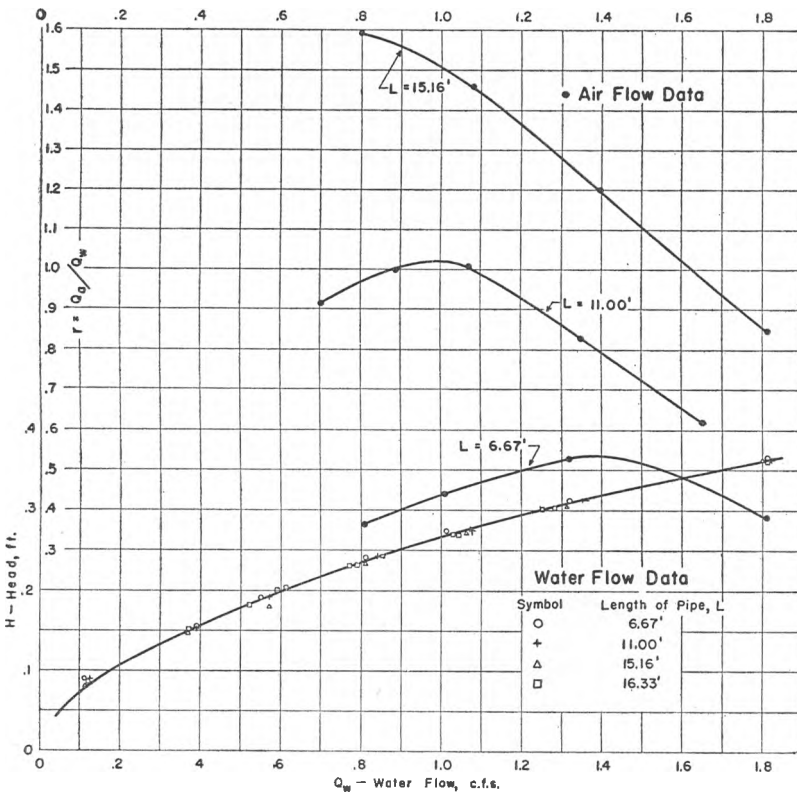


Fig. 3. Water and air flow data for 0.485 ft. drain pipe.

using still longer pipes. Present data indicate that the head-discharge relationship will be independent of pipe length for partly full flow. Also, for very long pipes, it appears that the critical head will tend to attain a fairly constant value.

The ratio, r , of the rate of air flow Q_a , to the rate of water flow Q_w , is also plotted in Figs. 2 to 7 inclusive. Note that a maximum value of r is reached at a relatively low head. However, the maximum value of Q_a is attained at a larger head. The rate of air flow tends to increase with the length of the pipe; however, it appears that the increase becomes less with increasing lengths and probably the rate of air flow will tend to attain a constant value for longer pipe lengths. There does not appear to be any significant difference for the air flows between overflow and drain pipes.

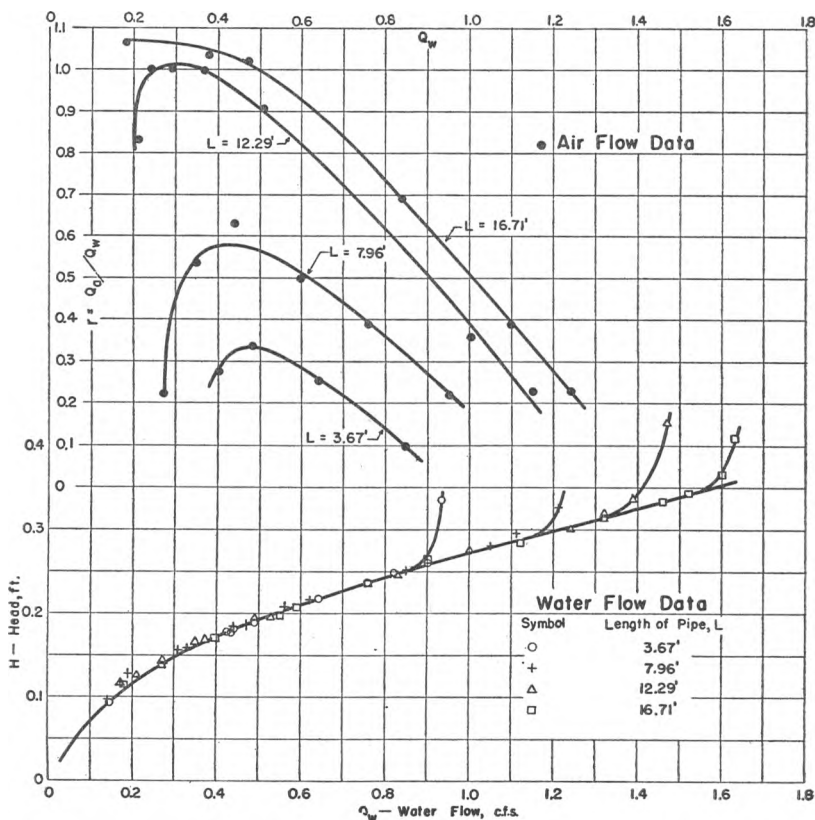


Fig. 4. Water and air flow data for 0.309 ft. overflow pipe.

For pipes about 2 or 3 ft. long, or less, it was possible, if care was used, to obtain perfect orifice flow with a solid jet of water falling straight down without touching the sides of the pipe. The water surface in the tank was perfectly smooth and no vortex tended to form. For longer pipes it was not possible to obtain this condition. The rate of flow under such conditions was considerably less for any given head than for the other type of flow. It seemed that the reason that this jet flow would not occur for long pipes was because the falling jet tended to draw along so much air that a high vacuum occurred near the top of the pipe. This partial vacuum caused the air above the water surface in the tank to push through and break up this jet flow.

For the normal conditions of flow a high vacuum occurred just

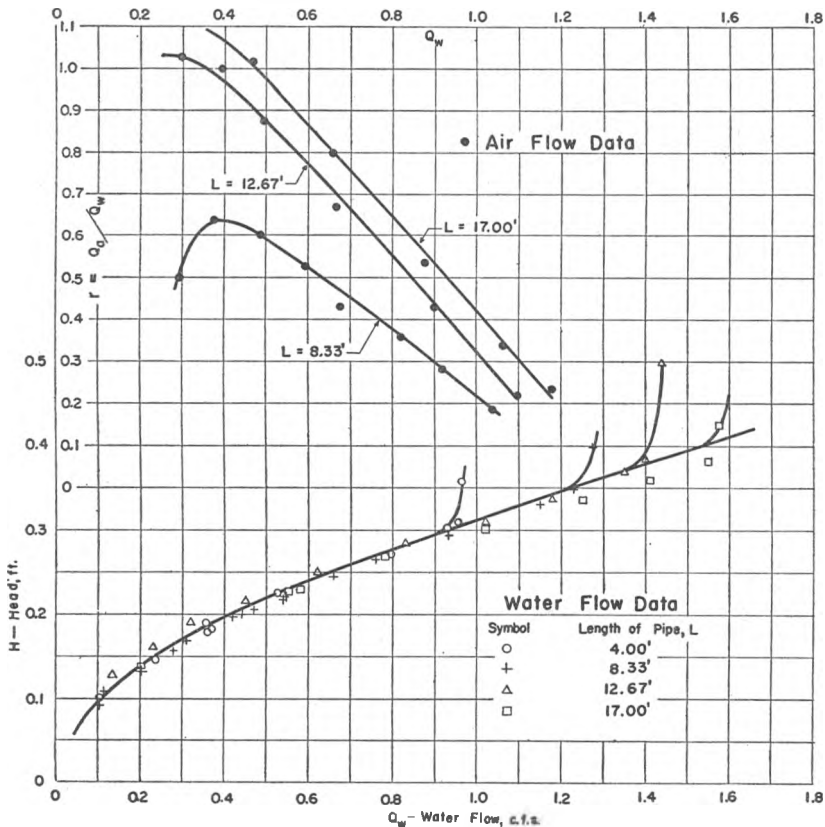


Fig. 5. Water and air flow data for 0.309 ft. drain pipe.

below the entrance, its magnitude increasing with discharge and length of pipe, and for a given discharge being considerably higher for the smaller pipes. For a water flow of just under 2 c.f.s. in the 0.485-ft. pipe a vacuum of 4 in. of mercury was measured for the longest pipe used. For a flow of about 1.3 c.f.s. down the 0.309-ft. pipe when it was 12.67 ft. long a vacuum of 12 in. of mercury was measured. This vacuum below the entrance and its dependence on the discharge, pipe size, and the pipe length had considerable influence on the particular head-discharge relationship obtained.

D. ANALYSIS AND CORRELATION OF DATA

It is of considerable interest and of practical usefulness to correlate the head discharge data for the various pipe sizes so as to

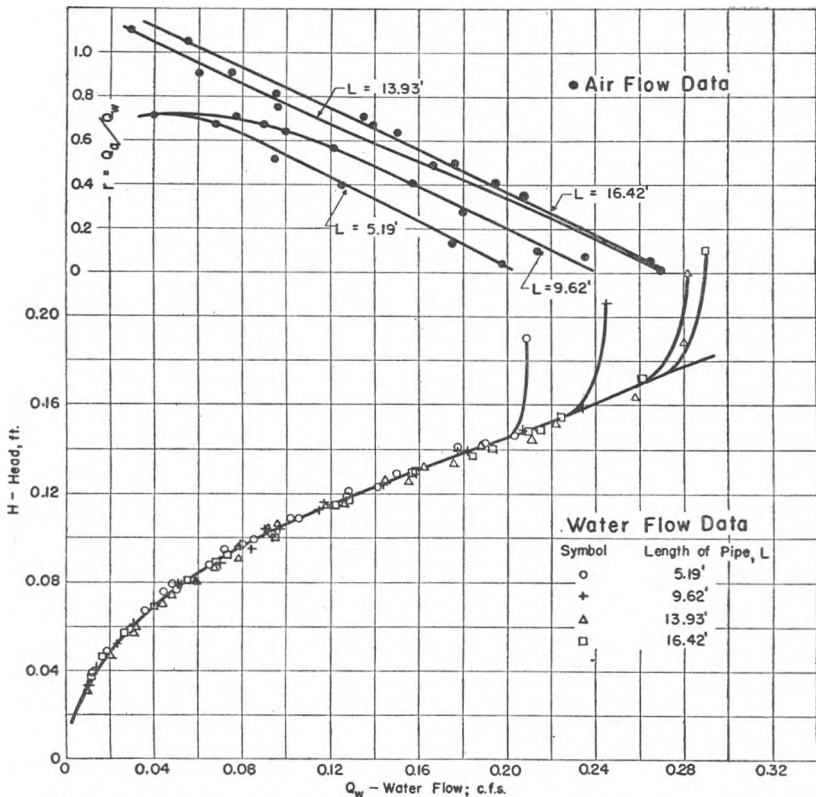


Fig. 6. Water and air flow data for 0.144 ft. overflow pipe.

permit convenient and accurate extension and interpolation of these data for other pipe sizes. Some attempts were made to develop a theoretical relationship; however, nothing useful was obtained. A plotting of the head-discharge data on log-paper indicated that for all the pipe sizes the discharge tended to vary as H^2 . Also, that for a given head, the discharge was proportional to $D^{1/2}$, that is, the square root of the pipe diameter.

One might hazard various guesses as to the reason for this particular head-discharge-diameter variation. It is probably tied up with the occurrence of a combination of weir and orifice flow, profoundly affected by the high vacuum formation below the entrance. The vacuum below the entrance undoubtedly causes a tremendous increase in discharge; however, it is peculiar that though longer pipes

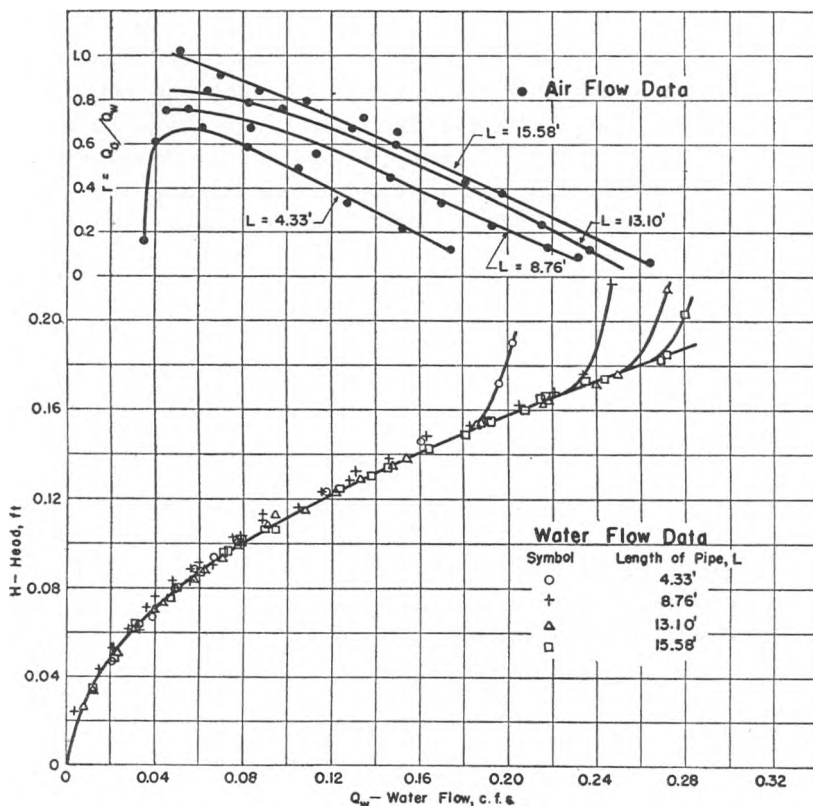


Fig. 7. Water and air flow data for 0.144 ft. drain pipe.

produced a higher vacuum the water discharge was not affected. Instead, the longer pipe and higher vacuum simply caused a higher air inflow.

The relationship that best correlates the data is:

$$Q_w = Cg^{1/2}D^{1/2}H^2 \quad (1)$$

The coefficient C is dimensionless and thus Eq. (1) can be considered as dimensionally homogeneous. Dimensional analysis indicated that C ought to depend on the ratio H/D . In Figs. 8 and 9 values of C for the various pipe sizes are plotted against H/D for the overflow and drain pipes respectively. The over-flow pipe values of C are also plotted for a 1-in. pipe from data obtained by Binnie.³ Though the data scatter somewhat, it is believed that the variation of C with H/D is significant and that Eq. (1) can be used for all practical purposes for calculating flow down overflow and drain pipes installed as described herein. Note that the value of C for overflow pipes is appreciably larger than for the drain pipes.

It should be pointed out that Eq. (1) is applicable only below the critical head, and therefore the critical head should be known before Eq. (1) can be safely used. An analysis can be made which should give information as to the approximate value of this critical head. When the pipe flows full the total head acting is equal to $(H + L)$, where L is the length of the pipe. The following expressions can then be set up:

$$H + L = C_e V^2/2g + (fL/D)V^2/2g + V^2/2g \quad (2)$$

where C_e is the entrance loss coefficient and f is the ordinary pipe friction factor. From Eq. (2) an expression for discharge can be developed:

$$Q = (K\pi D^2/4)\sqrt{2g(H + L)} \\ K = 1/\sqrt{C_e + (fL/D) + 1} \quad (3)$$

Graphically, the point where Eq. (3) and Eq. (1) intersect approximates the value of the critical head. On equating these expressions the following relation is obtained:

$$H/D = (K\pi\sqrt{2}/4C_e)^{2/3} (1 + L/H)^{1/3} \quad (4)$$

Therefore, in general the ratio H/D at the critical head for a given entrance condition is a function of L/D and L/H . The coefficients,

³ Binnie, A. M., "The Use of a Vertical Pipe as an Overflow for a Large Tank." *Proc. Royal Society of London*, vol. 168-A, p. 219, 1938.

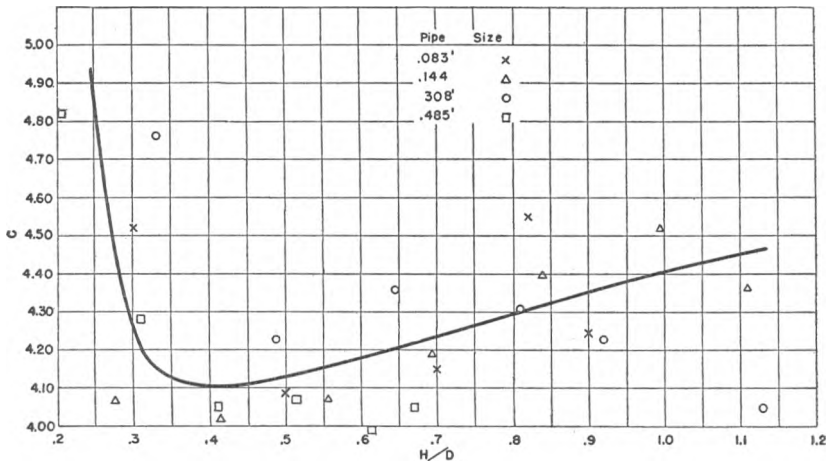


Fig. 8. Variation of water discharge coefficient for overflow pipes.

C and f are assumed constant; actually C is also dependent on L/H (See Figs. 8 and 9) and f varies with pipe roughness, diameter, and velocity of flow. In Eq. (3) the value of H is small compared to L ; therefore, neglecting H would not produce any great error. If now Eq. (3), with H omitted, and Eq. (1) are equated, the following relation results:

$$H/D = (K\pi \sqrt{2} / 4C)^{1/2} (L/D)^{1/4} \quad (5)$$

This indicates that the critical value of H/D is approximately a

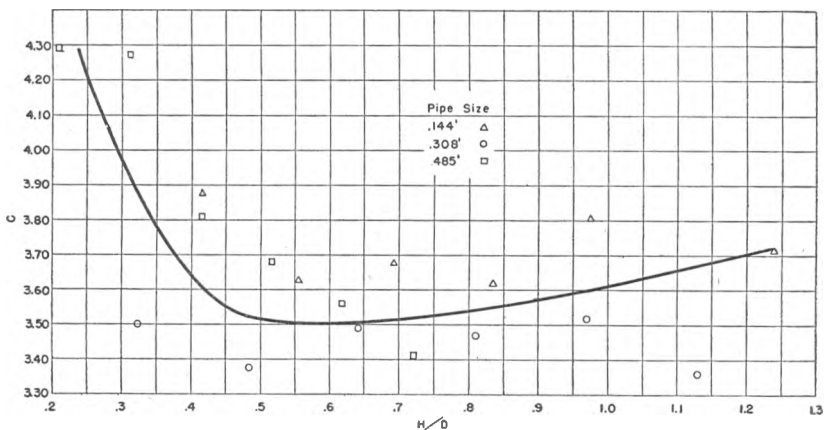


Fig. 9. Variation of water discharge coefficient for drain pipes.

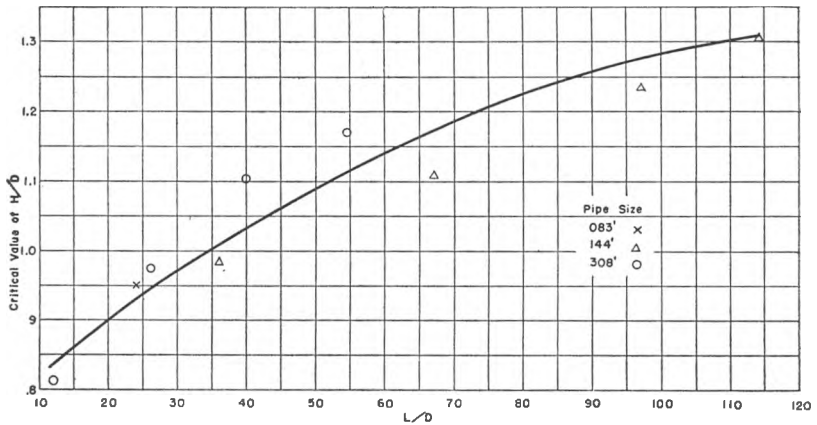


Fig. 10. Variation of critical head for overflow pipes.

function only of L/D . (It should be noted that K depends on L/D). Eq. (5) permits arriving at an approximate value of H/D , and then, if desired, a more exact calculation can be made using Eq. (4).

In Figs. 10 and 11 are shown plots of the critical value of H/D against L/D for the overflow and drain pipes respectively. The scattering of the points for the different pipe sizes indicates that some other factor is influencing the critical value of H/D . The factor that varies most for the different sizes of pipe is probably f , since for smooth pipes f varies considerably with the pipe diameter and velocity of flow. Another way to look at this problem is that the ratio L/D indicates geometric similarity and the ratio

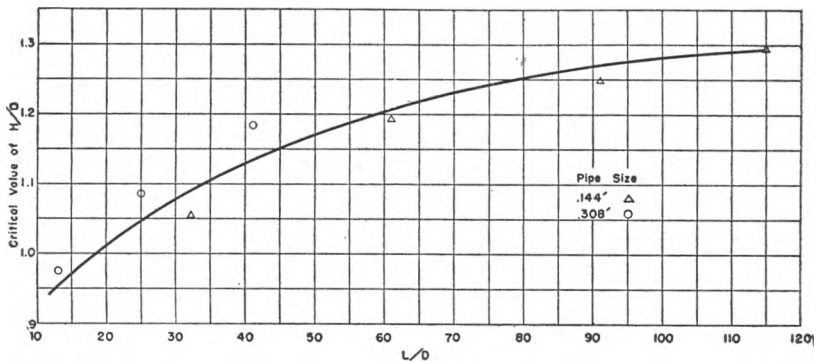


Fig. 11. Variation of critical head for drain pipes.

H/D is proportional to the Froude Number; however, in the plotting of Figs. 10 and 11 the Reynolds number was neglected. Though the Reynolds number does not have much influence when the pipe is not flowing full, it undoubtedly has an appreciable influence when the critical head is approached and the pipe begins to flow full.

Experiments were made to determine the value of the entrance loss coefficient, C_e , for the case of the pipe flowing full. The value of C_e for the case of the overflow pipe came out to about 0.70 and for the drain pipe entrance (flush connection) about 0.60. These values can be used for calculating \bar{K} in Eq. (3). The values of the critical value of H/D obtained by solving Eq. (5) check the actual experimental values quite well. It is, therefore, suggested that Eq. (5) be used instead of the more complex Eq. (4), which must be solved by trial and error.

E. SUMMARY AND CONCLUSIONS

Experimental data on the hydraulics and pneumatics of three sizes of vertical overflow and drain pipes when flowing partly full are presented. The head, which is measured above the top of the pipe, and the water discharge are correlated by a general expression which permits discharge calculations for pipe sizes other than those tested. It was found that the discharge varied as the square root of the pipe diameter and as the square of the head. For flow partly full the head-discharge relationship was independent of the pipe length. For a given pipe size and head the overflow pipe discharged slightly more water than the drain pipe.

The critical head above which the pipe starts to flow full was found to depend on the pipe size, pipe length, and type of entrance. An analysis was made which permitted the prediction of this critical head for any size and length of overflow or drain pipe. The predicted values checked the experimental data quite well.

The ratio of the rate at which air was drawn down with the water, to the water discharge, was a maximum at a relatively low head. The maximum rate of air flow for any given condition occurred at a head considerably less than the critical head. For any pipe size and water discharge the air flow increased with pipe length; however, the increase became less and less for longer pipes, thus indicating that for very long pipes the air inflow would tend to be independent of the pipe length.

ACKNOWLEDGMENTS

The financial aid for this investigation was given by the National Association of Master Plumbers. Mr. W. M. Wachter and Mr. Claude Lomax, graduate research assistants, carried out the experimental tests.

ABSTRACTS OF GRADUATE THESES

Design of a Dam on the Seyhan River, Turkey. ORHAN AKYUREK. M. S. Thesis, June 1940; Professor Lane, adviser. Design of a masonry dam consisting of overflowing and non-overflowing sections, taking into account earthquake effects. This dam forms a part of a proposed irrigation enterprise for the plains of the Seyhan River in Turkey.

Hydraulic Characteristics of a Navigation Lock with Floor Culverts. MILES M. DAWSON. M. S. Thesis, June 1939; Professor Mavis, adviser. An analysis of experimental data secured on a lock model built by the U. S. Engineers, indicating that filling and emptying can not be satisfactorily accomplished by the same culvert system and that the port area in the downstream half of lock should be less than in the upstream half.

Sediment Behavior in Upward Flow. WARREN DELAPP. M. S. Thesis, June 1940; Professor Rouse, adviser. Experimental and analytical study of variation in sediment concentration throughout suspensions produced by the upward motion of water through sand beds. (See p. 18.)

The Effect of Certain Fluid Properties Upon the Profile of the Hydraulic Jump. MORGAN D. DUBROW and JOHN C. GOODRUM. M. S. Thesis, June 1940; Professor Posey, adviser. The effect of surface tension and viscosity upon the profile of the hydraulic jump was investigated experimentally. Tap water and solutions of Aerosol were used in a six-inch flume and water, mixtures of water and glycerine, and kerosene in a two-inch flume. No well-defined trends were noted, but the profile of the jump seemed to become steeper as the viscosity increased. As an incidental result of the investigation, non-undular hydraulic jumps were observed for jump ratios of less than 2.0. When the jumps formed far enough downstream to allow the eddies which started in the head bay to converge toward the center of the flume, the profile became undular.

Practical Hydraulics in Highway Engineering. CARL F. IZZARD. M. S. Thesis, June 1940; Dean Dawson, adviser. Application of

hydraulic principles for improvement of culvert, drain, and ditch design.

Experiments on Waves in Rectangular Channels. V. A. KOELZER. M. S. Thesis, June 1939; Professor Mavis, adviser. Considerable work done on recording apparatus. Wave velocities checked Russell's formula within 10 per cent.

Reinforcement of Concrete Flume Corners. ORVILLE KOFOID. M. S. Thesis, June 1940; Professor Posey, adviser. Flume corners are subject to tensile stresses at the inside of the corners. Arrangements of reinforcing designed to resist this type of stress, tested by Waistlund and by Gumensky, were critically examined, and a new design developed. It was compared with the best design used in standard practice by tests of 16 corners with $3\frac{1}{2}$ -ft. or $4\frac{1}{2}$ -ft. legs. Simultaneous load and strain measurements were taken, and the new design was found to have greater ultimate strength and greater toughness than the standard design. Its strength at first crack, however, showed only slight superiority.

Studies on Runoff from River Bottom Lands. MARVIN O. KRUSE. M. S. Thesis, June 1940; Professors Lane and Howe, advisers. A study of actual field data on six drainage districts along the Illinois and Mississippi Rivers gave relations between seepage into the districts and river stages outside the levees.

Design of Outlet Works of the Han River Flood Control Reservoir. HSUAN KUO. M. S. Thesis, August 1939; Professor Lane, adviser. A design is worked out in this thesis for the outlet works of a retarding basin on the Han River in China which has been proposed to control the floods of this river for the protection of the city of Hankow and adjacent communities. The construction of this reservoir has been proposed by the Han River Conservancy Bureau, a Chinese governmental organization, and this thesis carries one step farther the preliminary design worked out by them. The plan consists of a diversion channel through a rock point, the channel being controlled by stoney gates. The river would be closed by an earth dam. Hydroelectric power would be developed at the dam and locks would be provided to take care of the shipping on the Han River.

A Comparison of Lacey's Stable Channel Relations with the Conditions in the St. Clair and Lower Mississippi Rivers. CHUNG-TEH LI. M. S. Thesis, June 1940; Professor Lane, adviser. For the purpose of this thesis two channels were selected which were known to be relatively stable. One of these was the St. Clair River, the outlet of Lake Huron, in which the flow has been practically constant for several thousand years. The channel of the Mississippi River just above the forks at the lower end has been also stable for a long time although the discharge in this section has varied over a wide range. An attempt was made to compare the cross sections of these streams with the cross sections which would be indicated for these conditions by the studies of Mr. Gerald Lacey, as a result of his investigations for irrigation canals in India. Neither of these sections of rivers showed close agreement with Mr. Lacey's theories.

Hydraulics of Culverts. A. R. LUECKER. M. S. Thesis, February 1939; Professor Mavis, adviser. Flow modifications caused by entrance conditions and submergence were determined.

The Spreading of a Water Jet on a Flat Floor. ENVER MURATZADE. M. S. Thesis, August 1939; Professor Lane, adviser. This thesis is the first of a series studying the spreading of a jet of water at super-critical velocities on a flat floor. The object was to obtain data for use in design of spillway chutes where it is desired to widen and thin the water stream at the entrance to a stilling pool.

Flow Transitions in Rectangular Channels with Super-Critical Velocities. WARREN E. WILSON. Ph.D. Thesis, August 1940; Professor Lane, adviser. This thesis is a continuation of the thesis on "Spreading of a Water Jet on a Flat Floor" and attacks the problem from the standpoint of wave analysis. The application of this theory to the flow is brought out and the process by which such conditions can be analyzed is outlined. The existence of unexpected vacuums along the sides of the channel have been indicated.

Chinese River Control During the 16th Century. FA YAO WONG. M. S. Thesis, June 1939; Professor Mavis, adviser. Translation and compilation of old manuscripts in the Chinese language.

REFERENCE LIST OF STAFF PUBLICATIONS

FISHWAYS

An Annotated Bibliography of Fishways. PAUL NEMENYI. *University of Iowa Studies in Engineering*, Bulletin No. 23, 1941.

An Investigation of Fishways. A. M. McLEOD and PAUL NEMENYI. *University of Iowa Studies in Engineering*, Bulletin No. 24, 1941.

HYDROLOGY

Maximum Discharges at Stream Measurement Stations Through December 31, 1937. G. R. WILLIAMS and L. C. CRAWFORD. *U. S. Geological Survey Water-Supply Paper 847*.

Analysis of Runoff Characteristics. B. S. BARNES. *Proceedings, A. S. C. E.* (Discussion of paper by Otto H. Meyer), p. 375, February, 1939.

The Structure of Discharge Recession Curves. B. S. BARNES. *Transactions American Geophysical Union*, Part IV, pp. 721-725, 1939.

A Report on the Water Requirements of Rice Irrigation. E. W. LANE, T. T. CHENG, and C. L. PIEN. *Journal of the Association of Chinese and American Engineers*, Vol. 20, Nov.-Dec., 1939.

Problems of Stream-Flow Forecasting on Tributaries of the Upper Mississippi. B. S. BARNES. *Bulletin American Meteorological Society*, March, 1940.

Flood Forecasting in the Upper Mississippi Valley. B. S. BARNES. *University of Iowa Studies in Engineering*, Bulletin No. 20, pp. 230-239, March, 1940.

Trends in Climatological Runoff Data for Mississippi River Basin above Keokuk, Iowa. L. C. CRAWFORD, V. A. KOELZER, and P. L. MERCER. *The Exponent*, p. 4, December, 1940.

Trend in Rainfall Record Confirmed. L. C. CRAWFORD. *Civil Engineering*, p. 45, January, 1941.

MECHANICS OF FLUIDS

Elementary Mechanics of Fluids. HUNTER ROUSE. Hectograph notes issued by Department of Mechanics and Hydraulics, 1940. Text for elementary undergraduate course paralleling Mechanics of Solids and Mechanics of Materials, providing basic principles of fluid motion as foundation for applied courses in the engineering departments.

MISCELLANEOUS

Conversion of Units and Dimensions. C. J. POSEY and J. W. HOWE. *Civil Engineering*, Vol. 10, p. 528, August, 1940.

Regarding Tree Bench Marks. L. C. CRAWFORD and W. I. TRAVIS. *The Exponent*, p. 16, May, 1941.

MODELS

Photographic Technique for Recording Direction of Surface Currents in Models. C. J. POSEY. *Civil Engineering*, Vol. 9, p. 619, October, 1939.

OPEN-CHANNEL FLOW

Backwater Curves in Theory and Practice. C. J. POSEY. *Proceedings of Hydraulic Conference, University of Iowa Studies in Engineering*, Bulletin No. 20, pp. 205-213, March, 1940.

PIPE FLOW

A New Method of Presenting Data on Fluid Flow in Pipes. A. A. KALINSKE. *Civil Engineering*, p. 313, May, 1939.

Solving Pipe Flow Problems with Dimensionless Numbers. A. A. KALINSKE. *Eng. News-Record*, p. 55, July 6, 1939.

PLUMBING

Safety Devices in Domestic Hot-Water Supply Systems. F. M. DAWSON and A. A. KALINSKE. *Plumbing and Heating Business*, p. 21, March, 1940.

Grease Interceptors. F. M. DAWSON and A. A. KALINSKE. *Plumbing and Heating Business*, p. 20, Nov., 1939.

Methods of Calculating Water-Hammer Pressures. F. M. DAWSON and A. A. KALINSKE. *Journal A. W. W. A.*, Vol. 31, p. 1835, November, 1939.

Hydraulics and Pneumatics of Plumbing Drainage System. F. M. DAWSON and A. A. KALINSKE. *Tech. Bulletin No. 2. National Assoc. of Master Plumbers*, 1939.

SEDIMENT TRANSPORTATION

Relation of Suspended to Bed Material in Rivers. A. A. KALINSKE and E. W. LANE. *Trans. A. G. U.*, pp. 637-641, 1939.

Laws of Transportation of Sediment by Streams: Suspended Load. HUNTER ROUSE. Report for Round Table Discussion on Role of Hydraulic Laboratories in Geophysical Research, National Bureau of Standards, September 13, 1939. Hectograph copies of report prepared for circulation by Iowa Institute of Hydraulic Research.

Criteria for Similarity in the Transportation of Sediment. HUNTER ROUSE. *Proceedings of Hydraulics Conference, University of Iowa Studies in Engineering*, Bulletin No. 20, pp. 33-49, March, 1940.

The Munch-Peterson Formula for the So-Called "Coastal Transportation Force." PAUL NEMENYI. *Trans. A. G. U.*, Part II, pp. 352-353, April, 1941.

Suspended Material Transportation Under Non-Equilibrium Conditions. A. A. KALINSKE. *Trans. A. G. U.*, pp. 613-617, 1940.

A Study of Sedimentation in a Miami Conservancy District Reservoir. E. W. LANE and J. C. KENNEDY. *Trans. A. G. U.*, p. 607, 1940.

The Different Approaches to the Study of Propulsion of Granular Materials and the Value of their Coordination. PAUL NEMENYI. *Trans. A. G. U.*, p. 633, 1940.

Notes of Limit of Sediment Concentration. E. W. LANE. *Journal of Sedimentary Petrology*, Vol. 10, pp. 95-96, August, 1940.

Sand Waves in the Lower Mississippi. E. W. LANE and E. W. EDAN. *Journal Western Society of Engineers*, p. 281, Dec., 1940.

TURBULENCE

Experimental Studies of Liquid Turbulence. A. A. KALINSKE. *Proc. of Hydraulic Conference, University of Iowa Studies in Engineering*, Bulletin No. 20, pp. 50-65, March, 1940.

Relation of Statistical Theory of Turbulence to Hydraulics. A. A. KALINSKE. *Trans. A. S. C. E.*, Vol. 105, p. 1547, 1940.

Relation of the Statistical Theory of Turbulence to Hydraulics. HUNTER ROUSE. (Discussion of paper by A. A. Kalinske.) *Trans. A. S. C. E.*, Vol. 105, p. 1567, 1940.

Relation of the Statistical Theory of Turbulence to Hydraulics. PAUL NEMENYI. (Discussion of paper by A. A. Kalinske.) *Trans. A. S. C. E.*, Vol. 105, p. 1581, 1940.

Investigations of Liquid Turbulence and Suspended Material Transportation. A. A. KALINSKE. *Fluid Mechanics and Statistical Methods in Engineering*, University of Pennsylvania Press, Philadelphia, pp. 41-54, 1941.

SYNOPSIS OF U. S. ENGINEER OFFICE REPORTS

Prepared under the direction of the District Engineer, U. S. Engineer Office, St. Paul, Minnesota by the U. S. Engineer Sub-Office, Hydraulic Laboratory, Iowa City, Iowa.

Laboratory Tests on Hydraulic Model of Lock and Dam No. 22, Mississippi River, Hannibal, Missouri. (Final Report No. 33), 77 pp., May, 1939. Tests on hydraulic models of Mississippi River Dam No. 22 were made to obtain data for stilling basin and Tainter gate hoisting machinery design and for gate operation. A total of 157 tests were made on 19 models. A gate sill with 6 ft. drop in 15.5 ft., and a stilling basin 40 ft. long with a single row of 7 baffle piers 3 ft. high, 17 ft. from the end of the apron, and a solid end sill 5 ft. high were found to be satisfactory.

Laboratory Tests on Hydraulic Model of Filling and Emptying System of the General Joe Wheeler Lock, Tennessee River, near Florence, Alabama. (Final report No. 34), 84 pp., July, 1939. Tests were made on a model of the General Joe Wheeler Lock to determine operating time, hydraulic system coefficients, and flow distribution. Limited, comparable data, obtained at the prototype, are presented to show the degree of similarity between model and prototype. The model structure, tests, and analyses are described, and illustrated with photographs and drawings.

Permeability Tests on St. Peter Sandstone Specimens. (Final Report No. 35), 49 pp., July, 1939. Permeability tests were made on four specimens of St. Peter sandstone obtained from the Ford sand mine near the Twin City Dam at four different elevations comparable to the foundation elevation of the proposed lower locks at St. Anthony Falls, Mississippi River, Minneapolis, Minnesota. The results indicate the variation in percolation rates in sandstone at the four elevations, in each of three mutually normal directions, and in samples having fracture planes of different magnitudes. In many tests, gradients were gradually increased until failure of the sandstone obtained. Permeability tests were made also

on sand obtained by crushing the sandstone. The data are presented in this report in graphical and tabular form. Comments based on observations during the tests are also presented. Difficulties encountered in evolving a test technique and the conditions that led to the adopted testing methods are explained.

Laboratory Tests on Hydraulic Models of Roller Gate Stilling Basins. (Final Report No. 36), 279 pp., August, 1939. This report covers comprehensive tests on models of non-submergible and submergible roller gates to determine shapes and dimensions of stilling basins adequate to protect Mississippi River dams operated under emergency conditions required in the discharge of ice from the pools. Water pressures on the roller gates and sills were also observed in the model for the purpose of determining water loads and chain pulls on prototype gates under various conditions of operation.

Laboratory Tests on Hydraulic Models of Submergible Tainter Gates. (Final Report No. 37), 271 pp., October, 1939. Part I — Stilling Basin Studies and Determination of Water Loads on the Submergible Tainter Gate. Tests made on models of submergible Tainter gates to determine the hydraulic characteristics of a truss-type and a drum-type gate when operated in various submerged positions are discussed in this report. Models were built to scale ratios of 1:28.87 and 1:4, model: prototype. The design of the proposed stilling basin was checked and discharge coefficients were determined for flow over the gates. Water pressures on the downstream faces of the submerged gates, and loads on the hoisting chains were determined. Aeration of the nappe, vibration tendencies of the gate, and the effect of extending the end shields above the water surface were also studied.

Part II — Tainter Gate Discharge Coefficients. The discharge capacities of several Tainter gate models were determined for gate openings ranging from 2 to 18 ft., under variable upper pool and lower pool water surface elevations. The effect on discharge capacity was noted for the following variations in the models: Recessed and non-recessed gate piers; angle Θ varying from 22° - $50'$ to 70° - $33'$; variations in the shape of the gate sill; stilling basins with and without baffle piers; various lengths of apron; and gate radii of 30 and 40 ft. Two nomographs were developed for empirical discharge coefficients, one for a Tainter gate with recessed gate piers

and submergible gate sill and the other for a Tainter gate on an ogee spillway.

Laboratory Tests on Hydraulic Model of Lower Approach to Proposed Landward Lock, Lock and Dam No. 2, Mississippi River, Hastings, Minnesota. (Final Report No. 38), 104 pp., December, 1939. A model of the Mississippi River in the vicinity of Lock and Dam No. 2, Hastings, Minnesota, in which a problem of navigation in the lower approach to a proposed new lock was studied, is the subject of this report. A number of approach channel schemes excavated in the existing right bank were studied to determine their effects on current conditions and on sedimentation in the navigation channel. The tests indicated that the most satisfactory arrangement was one in which the excavation was carried from the existing right bank of the river to a line prolonged from the right wall of the new lock, with a 500-ft. guard wall constructed as an extension of the river wall of the existing lock, and the left river bank straightened so as to approximately parallel the right bank.

Laboratory Tests on Hydraulic Model to Determine Roller Gate Coefficients for upper Mississippi River Navigation Dams. (Report No. 39 — Appendix to Final Report No. 13; see last paragraph on page 77, Bulletin 19), 72 pp., January, 1940. The tests described in this appendix were made on a model of the type of submergible roller gate installed in Mississippi River navigation dams 11, 12, 13, 14, 17, 18, and 21. The results of these tests are presented by a set of discharge curves for the range of conditions which will be encountered at these dams. Similar curves are included also to present in more usable form calibration data for other types of roller and Tainter gates in the Rock Island District.

Laboratory Tests on Hydraulic Model of Lock and Dam No. 11, Mississippi River, Dubuque, Iowa. (Final Report No. 40), 111 pp., April, 1940. Part One of this report describes model tests made to indicate the best combination of gates and overflow spillway sections to be included in Dam No. 11, and the most suitable modifications of the piers and approach of the highway bridge immediately downstream. Part Two describes tests made on a hydraulic model of the structure to indicate the most desirable dis-

tribution of flow through the gates of the dam as constructed, with the objectives of reducing scour in the river bed which might endanger the stability of the structures and of avoiding the deposition of material in the navigation channel.

Laboratory Tests on Hydraulic Model of Diversion Channel and Stilling Basin, Dry Run Flood Control Project, Decorah, Iowa. (Final Report No. 43), 37 pp., October, 1940. The Dry Run diversion channel is a proposed project for flood protection at Decorah, Winneshiek County, Iowa. Tests on a model of the diversion channel were made to observe the performance of the channel as designed, and to develop a stilling basin for the outlet. A stilling basin 75 ft. long, 130 ft. wide, with a solid end sill 9 ft. high, was found to be satisfactory. This report is the preliminary presentation of the test data and the conclusions drawn from them. It contains the information essential to an understanding of the prototype problem, describes the model arrangement, gives the procedure of the tests, and conclusions with recommendations on hydraulic features of the design.

Laboratory Tests on Hydraulic Model to Determine Navigation Conditions in Approaches to St. Anthony Falls Locks, Mississippi River, Minneapolis, Minnesota. (Final Report No. 44), 143 pp., November, 1940. The St. Anthony Falls Navigation Project requires the construction of a new dam, consisting of three submergible Tainter gates; two locks, one with a lift of 50 ft. at St. Anthony Falls and the other with a lift of 25 ft. at the new dam; guide fences; a dredged navigation channel; and alterations to certain existing bridges. A hydraulic model of this project was built and tested to determine the navigation conditions in the Mississippi River and the lock approaches, to study the operation of the proposed locks and dam, and to develop any corrective measures found necessary. In general, the project was found to be entirely feasible. Possibly the most valuable result obtained from the model was the successful development of a training wall in the middle pool to eliminate dangerous cross currents in the navigation channel and lock approaches.

Laboratory Tests to Determine Discharge Coefficients for Obstructions to Super-Flood Flows. (Final Report No. 45), 55 pp., December, 1940. This report describes a model study con-

ducted for the purpose of determining coefficients of obstruction for bridges subject to floods which would result under the most critical combination of heavy storm run-off and saturated ground conditions that could occur on any watershed in the Rock Island District. Chapter II describes the basic model and accessories, Chapter III the testing procedure in the general studies, including the computation methods and results obtained from the bridge coefficient studies, and Chapter IV describes a series of tests in which existing conditions in the vicinity of the junction of the Des Moines and Raccoon Rivers at Des Moines, Iowa, were approximately simulated. The water surface profile of the Des Moines River, the bridge coefficients determined for this set-up, and the backwater effect of the Scott Street bridge and dam are included.

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STUDIES IN ENGINEERING

Bulletin 1. The Flow of Water Through Culverts, by D. L. Yarnell, F. A. Nagler, and S. M. Woodward, 1926. 128 pages, 26 figures, 23 plates. Out of print.

Bulletin 2. Laboratory Tests on Hydraulic Models of the Hastings Dam, by Martin E. Nelson, 1932. 72 pages, 40 figures. Out of print.

Bulletin 3. Tests of Anchorages for Reinforcing Bars, by Chesley J. Posey, 1933. 32 pages, 18 figures, price \$0.50.

Bulletin 4. The Physical and Anti-Knock Properties of Gasoline Blends, by Theodore R. Thoren, 1934. 32 pages, 13 figures, price \$0.35.

Bulletin 5. The Transportation of Detritus by Flowing Water—I, by F. T. Mavis, Chitty Ho, and Yun-Cheng Tu, 1935. 56 pages, 15 figures, price \$0.50.

Bulletin 6. An Investigation of Some Hand Motions Used in Factory Work, by Ralph M. Barnes, 1936. 60 pages, 22 figures, price \$0.60.

Bulletin 7. A Study of the Permeability of Sand, by F. T. Mavis and Edward F. Wilsey, 1936. 32 pages, 12 figures, price \$0.35.

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Bulletin 20. Proceedings of Hydraulics Conference, edited by J. W. Howe, 1940. 260 pages, 84 figures, price \$1.00.

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Bulletin 25. The Electrostatic Effect and the Heat Transmission of a Tube, by Melvin R. Wahlert and Huber O. Croft, 1941. 40 pages, 10 figures, price \$0.40.

REPRINTS

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Reprint No. 3. Flow of Water Around 180-Degree Bends, by David L. Yarnell and Sherman M. Woodward, 1937. 64 pages, 48 figures, price \$0.10. A reprint of Technical Bulletin No. 526, United States Department of Agriculture, Washington, D. C., Oct. 1936.

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An Analysis of Unusual Precipitation Records in Iowa, by F. T. Mavis and J. W. Howe. Reprinted from Journal of the American Water Works Association, v. 27, no. 2, Feb. 1935.

The Frequency of Intense Rainfall in Iowa, by F. T. Mavis and D. L. Yarnell. Reprinted from the Bulletin of the Associated State Engineering Societies, Oct. 1935.

Fundamental Hydrologic Considerations for the Design of Impounding Reservoirs in the Middle West, by E. L. Waterman, F. T. Mavis, and Edward Soucek. Reprinted from Journal of the American Water Works Association, v. 28, no. 2, Feb. 1936.

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Flush Wave Velocities in Sewers, by E. W. Lane and O. J. Baldwin. Reprinted from Engineering News-Record, June 11, 1936.

Sutro Weir Investigations Furnish Discharge Coefficients, by E. Soucek, H. E. Howe, and F. T. Mavis. Reprinted from Engineering News-Record, Nov. 12, 1936.

Predicting Stages for the Lower Mississippi, by E. W. Lane. Reprinted from Civil Engineering, Feb. 1937.

Reprint No. 6. Sewage Treatment at Iowa City, Iowa, by Earle L. Water-

man and Royal E. Rostenbach. Reprinted from Sewage Works Journal, v. 10, no. 1, pp. 106-114, Jan. 1937. Price \$0.15.

Reprint No. 7. *Miscellaneous Papers on Management.*

A Plan for Job Evaluation, by Ralph H. Landes. Presented at Management Conference in Iowa City, April 8, 1938.

Practical Applications of Motion-Study Research, Ralph M. Barnes. Reprinted from Mechanical Engineering, May, 1938.

References to Papers by the Other Conference Speakers.

Reprint No. 8. Hydraulic Jump in Trapezoidal Channels, by C. J. Posey and P. S. Hsing. Reprinted from Engineering News-Record, Dec. 22, 1938.

Hydraulic Jump in Closed Conduits, by E. W. Lane and C. E. Kindsvater. Reprinted from Engineering News-Record, Dec. 29, 1938. Price \$0.10.

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Solving Pipe Flow Problems With Dimensionless Numbers, by A. A. Kalinske. Reprinted from Engineering News-Record, v. 123, no. 1, p. 55, July 6, 1939.

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Engineering Aspects of Sediment Transportation and Deposition, by E. W. Lane. Reprinted from Bulletin of Associated State Engineering Societies, Oct., 1939.

Collection of Data on the Solids Load of Flowing Streams, by E. W. Lane. Reprinted from Journal of the Association of Chinese and American Engineers, v. XIX, no. 3, May-June, 1938.

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The Hydraulics and Pneumatics of the Plumbing Drainage System, by A. A. Kalinske. Reprinted from Bulletin of Associated State Engineering Societies, Oct., 1938.

The National Plumbing Laboratory, by F. M. Dawson and A. A. Kalinske. Reprinted from American Journal of Public Health, v. 30, no. 1, Jan., 1940.

Plumbing Free From Pollution, by F. M. Dawson. Reprinted from The Nation's Schools, v. 23, no. 5, May, 1939.

The Prevention of Back-Siphonage, by F. M. Dawson and A. A. Kalinske. Reprinted from Plumbing and Heating Business, Sept., 1939.

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A Study of Variability of Precipitation, by Edward Soucek and J. W. Howe. Reprinted from Transactions of the American Geophysical Union, Nineteenth Annual Meeting, 1938.

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